EFFECT OF VIBRATIONS ON COMPRESSIVE STRENGTH OF COHESIVE SOILS

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by

B.K.Sahu

DEPARTMENT OF CIVIL ENGINEERING
INDIAN INSTITUTE OF TECHNOLOGY
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The study has been made in two parts. In part I, the frequency-displacement study of cylindrical specimens of the cohesive soil has been made. The variables include frequency of vibratory loading, moisture content, density and size of the specimen. The investigations are confined within the range of 0 to 30 cps. In the frequency-Displacement pattern two peaks are observed, which correspond to the resonant frequency. It is observed that resonant frequencies in both cases i.e. with samples of diameter 1.5 in. and height 3.375 in. as well as with samples of Diameter 4.0 in. and height 4.5 in. lies in the same range.

In part II, the variation of unconfined compressive strength with frequency under vibratory loading has been studied. The variables considered include the frequency of the vibratory loading, moisture content and density. It is seen that strength is minimum at the resonant frequencies.

Such tests are very useful for finding the effect of vibration on the compressive strength and its modolus especially near the resonant frequency range.

INTRODUCTION

During the past 10 to 15 years there has been a marked increase in the study of soil behaviour under dynamic loading. On the basis of a recent inventory made by the Civil Engineers it has become one of the main topics of current research. There have been various motivating reasons, the following appearing to be more important.

1. Realization by Highway Engineers that the development of rational methods of pavement design requires the consideration of the behaviour of subgrade, the base courses and the wearing surface under the action of transient repeated stress.

2. The appearance of new dynamic problems in connection with the structural requirements of facilities for launching or operating spacecraft, missiles and heavy weapons.

3. The need to study the effect of nuclear and other very powerful blastings.

4. The ever present interest in the behaviour of soils under the action of repeated stresses produced by seismic action and machine foundations.

5. The necessity to investigate slope stability of regions which are continuously subjected to seismic effect.

The dynamic loads acting on soils and foundation in the field are of various characters. The well known example of a machine foundation is one in which the load is applied repeatedly in quick succession. The frequency

of the operating system, mode of transference of the vibratory load, foundation soil interaction are of great importance.

Embanknents subjected to dynamic loads e.g. earthquakes and bombblasts, is another example. The loading is of transient nature. The number of repetitions of impulsive (transient) load is dependent upon the nature of earthquake and sequence of blasts. The deformations of ground on account of such loads are not systematic and take place in all directions. Artificial loading e.g. quarry blasting is similar to a bomb blast or a miniature earthquake.

Heavy wheeled traffic over pavements is of the slow repetitional type and depending upon the rapidity of load applications, it may constitute a dynamic load.

Both soils and foundations have been tested in laboratory by various investigators to determine vibratory characteristics of soils and foundation soil interaction. The tests on soils were primarily of two types. In one type of tests, the soil samples were subjected to vibratory loading of known characteristics i.e. stress level, frequency and amplitude of vibration and the samples were later sheared as in routine tests (Balkrishna Rao 1961) while in the other tests the soil was under constant vibration when under tests (Mogami and Kubo 1953)

Kondner(1961). The loading conditions in the latter tests simulates field loading to a better degree.

The tests under transient loading were also of two types. In one type of tests the load was applied in single transient pulse, (Casagrande and Shanon 1949). Whitman (1957a,1962) while in other tests the transient pulses were repeated at regular short intervals, a number of times (Seed 1960 a). In transient testing of soils, a combination of static and dynamic loads simulates the field conditions to a better degree.

The data on dynamic behaviour of soil s reported in published literatures does not make a coherent picture and at best is invariably qualitative in nature. Sometimes, the results reported by different investigators tend to disagree.

In chapter II, all the pertinent literature available on the subject has been briefly and critically reviewed.

In chapter III, experimental set up, developed for the vibratory test at I.I.T.Kanpur, and the method of conducting the test has been discussed, and chapter IV contains the discussion, result and the conclusion of the research done on vibratory behaviour of cohesive soils.

In the present work attempt has been made to study the possibility of testing soils under constant vibration when under test (conventional unconfined compressive strength test). Thus the loading conditions simulate field loading to a better degree. The variables considered include frequency, density and moisture content.

LITERATURE REVIEW

A French Engineer, A.Collin first recognised the time dependent nature of soil strength in 1846. Reference was made to "instantaneous" and " permanent" soil strengths. These implied respectively the resistance to temporary forces with a duration less than 30 Sec. and to permanent forces not significantly attend after a considerable lapse of time. Collin used a double-shear device and observed that the permanent strength of clay may be in the range of 24% to 34% of the instantaneous strength. As a result of this work, Collin emphasized the importance of accurately evaluating the load duration as well as its magnitude. Collin also said "knowledge of the absolute instantaneous resistance is of no use in construction practice". For many years, only the "permanent" strength (long-term stability and creep problems) of soils received the attention of investigators.

However, recent developments in technology require a better understanding of the dynamic behaviour of the soils.

The papers to be reviewed are grouped under two headings:-

(a) Transient loading

(b) Vibratory loading

Transient loading:-

Casagrande and Shannon conducted certain soil dynamic investigations in 1948. They reported the results of tests on one sand and three clays. The tests were performed in a

Triaxial machine in which time to failure under single impulse transient loading was one hundredth of a second. It was found that the transient strength of clays for the fastest test was from 1.5 to 2.0 times the static strength. The strength of sand increased only slightly, maximum increase in the fastest being 10%.

In another series of triaxial compression test carried out to determine the strength of a very soft organic clay, failure was produced within a range of 1.7 to 7.0 hours. These tests indicate that the strength at the fastest rate of loading was about 40% greater than at the slowest rate.

Seed and Lundgren reported tests on a fine and coarse sanc. The tests were performed in a triaxial shear machine with lateral confinning pressure of 2 Kg/cm^2 in all tests. Three types of tests were performed on both of these sands. These are (1) Static tests in which time of loading to failure was 10-15 min. (2) Slow-transient tests at constant rate of deformation of 6 in per minute with time of loading to failure of about 4 seconds and (3) rapid transient tests at constant rate of deformation of 40 in / Sec. in which the time of loading to failure was about 0.02 sec. The size of specimen in each case was 1.4in. diameter and 4.0in.in height. Their data indicated that (a) strength of dense sands in undrained static test was about 20% greater than that in the drained test.

(b) The strength obtained in rapid transient test, both drained and undrained was the same i.e. the rate of loading was so fast that there was no time for drainage to occur.

(c) The static and slow transient strengths in undrained tests are same i.e. rate of loading upto time of 4 seconds does not constitute any significant change in strength.

(d) In rapid transient tests about 20% strength was increased due to dilatency as was the case in drained and undrained tests and another 15 to 20% strength increase is due to rate of loading.

Whitman(1957a) performed triaxial tests on 1.4 inch diameter and 3.5 inch long samples both confined and unconfined. The failure time was upto 0.001 sec. under single impulse transient loading.

From tests on four cohesive soils, it was concluded that:-

(1) The strength of unconfined samples depended on strain rate to a larger degree as a compared to confined samples.

(2) In unconfined samples, which failed either by splitting or by developing shear planes, the peak resistance occured,at a much larger strain in rapid tests than in slow tests.

(3) In confined samples, the strain required to develop peak resistance was almost independent of the time of loading.

Tests on uniform ottawa and a well graded dry sands showed that their compressive strength increased about 10-15% between times of loading of several seconds and 0.05 seconds.

These findings are in agreement with similar results reported by Casagrande and Shanon (1948 and 1949). No significant increase in strength accured as the time of loading was decreased to 0.005 secs. The strength characteristics exhibited by moist sands were similar to those of dry sands.

Transient pore water pressure was recorded during tests on saturated sands. Data obtained from such tests showed that the pore water pressure at a given strain was much smaller in rapid tests than in slow tests.

A few transient tests with pore pressure measurements were performed on loose samples. There was no evidence of liquefaction.

Whitman(1957b) reported tests on a coarse uniform and a fai~~r~~ly well graded sand, performed by Soteriades (1954). The strength was found to increase by about 10% between strain rates of 0.0025% and 5% per second. His creep and relaxation tests suggested that approximately 15% resistance change would occur, if the time to failure would be reduced from several seconds down to 0.1 second. Work at M.I.T. indicated that no further significant strength increase occured, until the loading time is so short that inertia of the grains becomes important.

An explanation for increased strength of dense sands in rapid undrained tests as compared to slow undrained tests is offered, based on pore pressure gradients which are set up

during such tests on saturated sands. This does not conform to the assumption of Seed and Lundgren (1954) according to whom, the pore pressure changes in undrained tests were the same regardless of the strain rate. This may not be correct since in rapid undrained tests, the water can not possibly migrate from one portion of the sample to the other quickly. An indirect proof of this statement lies in the results of rapid transient tests (Seed and Lundgren 1954) in which the strength in drained and undrained conditions was the same. In very soft, plastic soils and in confined samples the strain at failure is independent of the strain rate.

Seed (1960 a) performed tests on confined samples of silty clay, prepared by compaction, in a triaxial machine. The specimen was loaded to a triaxial machine. The specimen was loaded to 66% of failure load (deviator stress in a static test) i.e. F.S.=1.5 and allowed to come to equilibrium over a period of 30 minutes. Then a series of 100 transient load applications, corresponding to an initial stress change of 35% were applied. The data led Seed to conclude that:-

(1) Soil strength under the simulate earthquake loadings is appreciably less than that indicated by transient tests, but,

(2) The strength is considerably more than that indicated by long term vibratory tests.

Seed (1960) reported transient triaxial tests on two sands, Ottawa sand and a camp Cooke sand. The sands were tested in dry and saturated state and in a dense as well as loose state. The rates of loading were 0.012 in./sec. 0.08 in /sec. and 18 in /sec. For dry sand it was concluded that a variation in friction angle, in the range of 5 minute to 5 milliseconds of time to failure, is no more than 10% and is probably less than 5%. The peak strength of dense saturated Ottawa sand in undrained tests for rates of loading of 0.08 in /sec. and .18 in/sec. was the same. The tests of Seed and Lundgren (1954) indicated an increase in strength of the order of 20% when the rate of loading was increased from 6 in./min. to 40 in/sec. are apparently not in agreement with Whitman.

Test on Ottawa sand in loose saturated state, tested at rates of strain of 0.08 in/sec. indicated initial yielding at an axial strain of 0.5%. At larger strains, the sand tended to dilate more when strained rapidly than when strained slowly. The strength defined as the axial stress at 20% strain increased by 40% in the rapid test. For the Camp Cooke sand, the increase amounted to 100%.

VIBRATORY LOADING:

Bendel (1948) reported tests on a loan at water content of 22%. The size of cylindrical samples for triaxial tests was 20 sq. cm. and 10 cm. high. The deformation of the

sample was measured with the help of change in resistance of a 0.025 mm. wire pasted on to a thin paper in the form of manifold loops.

The samples were subjected to all round pressure of 1.5 K gm/cm^2 for a number of days and then subjected to dynamic strains in which the frequency of loading varied from 0 to 20.5 cps.

It was observed that at the beginning of the dynamic stress application the sample underwent a sudden deformation of 2 mm. and continued to deform plastically thereafter. At the excitation frequency of 21 cps the sample completely failed by collapse.

Mogami and Kubo (1953) presented tests on sand and loam which were filled into a metal box fixed on an electrical vibration table oscillating vertically and sinusoidally. Maximum vibration amplitude was 2 mm. and frequency from 10 cps to 50 cps (800 $cm/sec.^2$).

They found that shearing strength decreased with acceleration as a function of increasing acceleration , in a very striking manner. In fact it decreased almost to zero which could justify liquefaction. They further concluded that the density of the Kumino sand was altered by loading.

Mogami, Yamoguchi and Nakase (1955) reported direct shear tests on undisturbed and remoulded clay with natural m/c of 110%, LL=110% and PL=50%. The vibration amplitude and frequency of vibration in tests, was the same as in above tests.

The specimen of remoulded clay were vibrated for 5 minutes or 1 minute before shearing, the acceleration of vibration being 980 cm/sec^2 and frequency 48 cps. The effect of vibration on shearing strength was negligible which was attributed to insufficient duration of vibrations and disturbance of soil due to hammering action.

In another series of tests on undisturbed samples, stress, smaller than failure stress, was applied to the soil sample and it was then vibrated. The strength of soil was reduced to 98% of the static strength when vibrated ~~xxxxx~~ at acceleration of 980 cm/sec^2 and frequency of 48 cps.

They further performed similar tests on remoulded soil samples vibrated with acceleration of 2 g and g/2 at frequencies of 48,30 and 24 cps. at a variable amplitude. They found that shear strength of soil samples depends in a characteristic manner on the magnitude of acceleration. Mencel and Kazda(1957) presented conclusion of L'Hermite and Tournon (1948) box shear tests according to which the strength during vibration had sunk nearly to zero, while on the other hand Krey (1936) had observed that natural slope at vibrations was only $1\frac{1}{2}^{\circ}$ smaller than the one at rest.

According to Mencel and Kazda, the apparently conflicting results are caused by the fact that in each

different case an individual change of normal stresses occur. Both stresses can originate from different phases as there may be a possibility for normal stress to diminish at the same time when the shear grows.

Murayam and Shibate (1960) investigated the effect of vibrations on shearing strength of un-disturbed clay in a direct double shear box having two mass oscillator to give vertical load to the specimen. The frequency range from 576 to 2970 cpm. The soil had a clay fraction of 50%, LL 67.3%, PL 27.3% with a natural water content of 52.3% and a preconsolidation stress of 1.8 Kg/cm^2 . The samples were cut in a prismatic shape 4 cms. square and 5.5 cms long.

Shearing strength was measured by increasing the horizontal shearing force continuously at a suitable rate until the shear failure occured. They showed that the shearing strength decreases linearly with increase in maximum acceleration of vibration.

Kondner (1961) presented test data for clay sample having L.L. 42%, PL = 21% and sp.gravity = 2.68. The samples were prepared by impact compaction process, the wet density being 131 Lbs/c ft at water content of 18.7%. A constant frequency of 25 cps was used while testing the sample.

Two type of box shear tests were conducted. In one , the amplitude of shear deformation was varied and the corresponding shear forces were measured for constant frequency of vibration. In other tests the specimen were subjected to shear deformation of constant amplitude but with different frequencies of oscillation. The shear force required to maintain constant shear deformation was measured with changes in frequency.

It was found that the strength of clay under vibratory loading is considerably less than that under static loading alone. It showed that the elastic response of clay is definitely non-linear.

Kondner (1962) performed vibratory unconfined compression tests on the same clay as in the above tests at m/c of 22.7% with wet density of 127 Lbs per cft. The frequency of load application was 25 cps. He found that the maximum stress required to cause failure was 35.2 psi. for the conventional tests while that of vibratory tests was only 8 psi, thus the strength under vibratory loading was only one fourth of that obtained by the conventional tests. Results of other tests indicated that strength under vibratory loading was only one half to one fifth of that obtained by conventional unconfined tests depending upon the static stress level, the moisture content and nature of vibratory loading .

He further performed the similar vibratory tests in which samples were subjected to vibratory deformation of constant

amplitude but with different frequencies of oscillations. He plotted the results in terms of complex compression modulus $E(\sigma_D / \epsilon_D)$ Vs frequency. Inspite of so many suspicious fluctuations, the complex compression modulus tends to decrease with frequency. For the range of tests (5 to 200 cps) conducted on clay under consideration, he has concluded as follows:

1. The strength of clay under vibratory loading is considerably less than under static loading alone. These reduction in strength seem to be caused by changes in the energy state of the bound water phase of the soil.
2. The elastic response of the material is definitely non-linear.
3. The dynamic response is a function of the static stress level about which stress perturbations are taking place.
4. The response of the material is frequency dependent.
5. Extreme variation in specimen size influence the results of the testing methods considered.

Balkrishna Rao (1961-1962) presented test data of unconfined compression tests on a red soil, in which the specimen was vibrated under steady vibrations and then tested. A Degebo type operated by a D.C.Motor was used with range of frequency vibrator from 350 rpm. The samples were $1\frac{1}{2}$" in diameter and 3" high. He concluded that if a standard sample specimen exhibits a very high strength; the effect of vibration is to decrease the strength and on the other hand if standard sample exhibits lower strength, the effect of

vibration is to increase the strength . Secondly the load carrying capacity of standard and vibrated specimen was found to be a function of total deformation that specimen could take before failure.

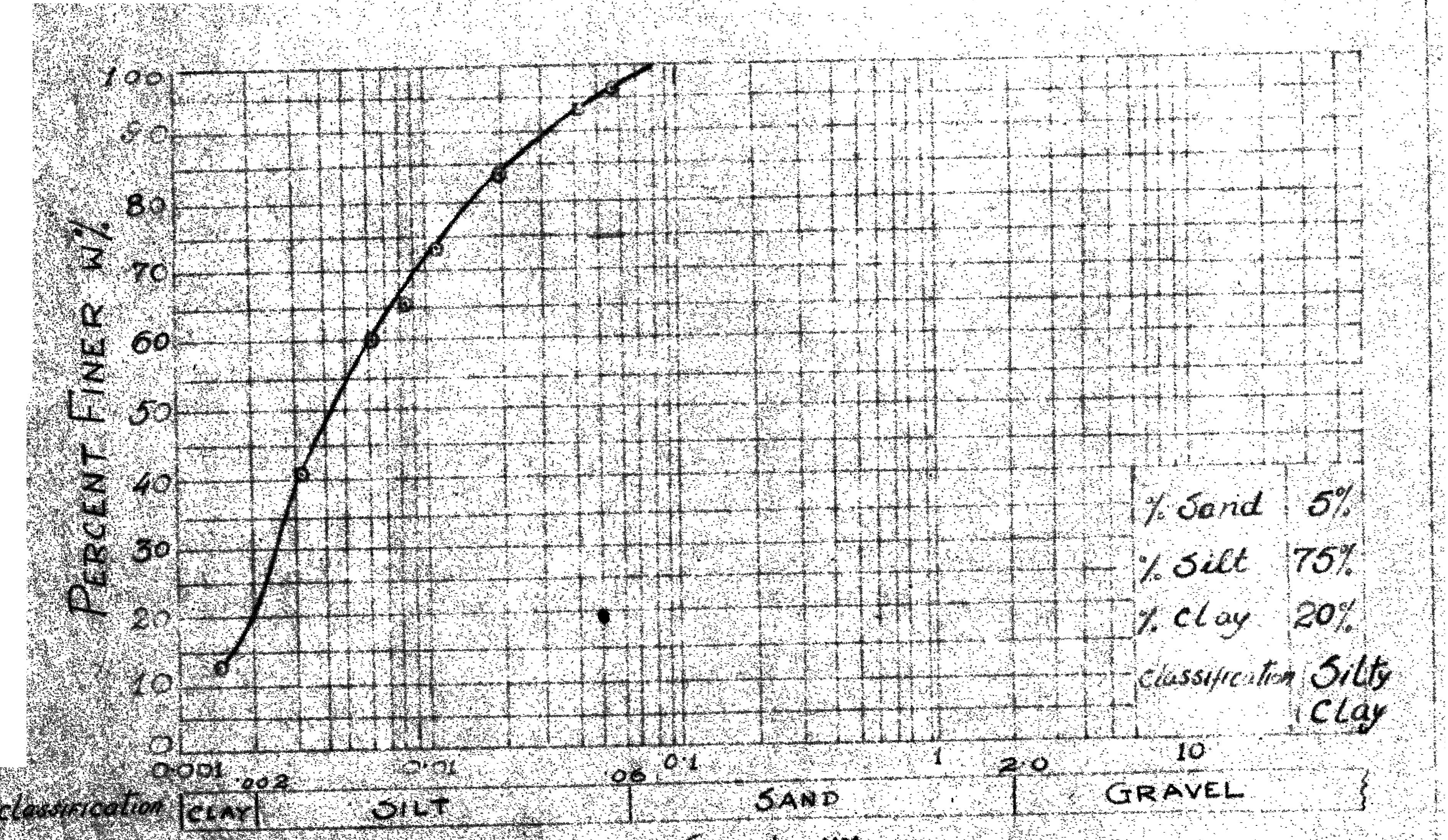
Wolkfill and Buchanan (1962) reported results of triaxial tests on 6" diameter and 12" high samples of Edward lime stone. Before the conventional triaxial test at a rate of 0.05 in/min., the sample had been subjected to 5,000 and 239,000 repetitions of stress application with leteral pressure of 60psi, the stress level in the vibratory loading was 50% of the ultimate strength and the rate of loading averaged 27000 psi per second. The increase in strength was 12% after 5000 repititions and 20% after 239,000 repititions. Data on other tests similar to these is also reported in which the lateral pressure and number of repititions of vibratory load was varied. The conclusion of Mogami, Yamaguchi and Nakase (1955), Balkrishna Rao (1961-62) and Wolkfill and Buchanan (1962), in respect to the effect of vibrations on the strength of soils are in apparent conflict with each other.

J.Foloque(1965) conducted dynamical triaxial tests on compacted unsaturated soils. Behaviour under hydrostatic pulses and under deviator stresses was studied. He concluded that volume strains were practically unaffected by the frequency of applied pulses. Deviator strains, for loads applied during

same time increases significantly with the frequency of applied deviator stresses.

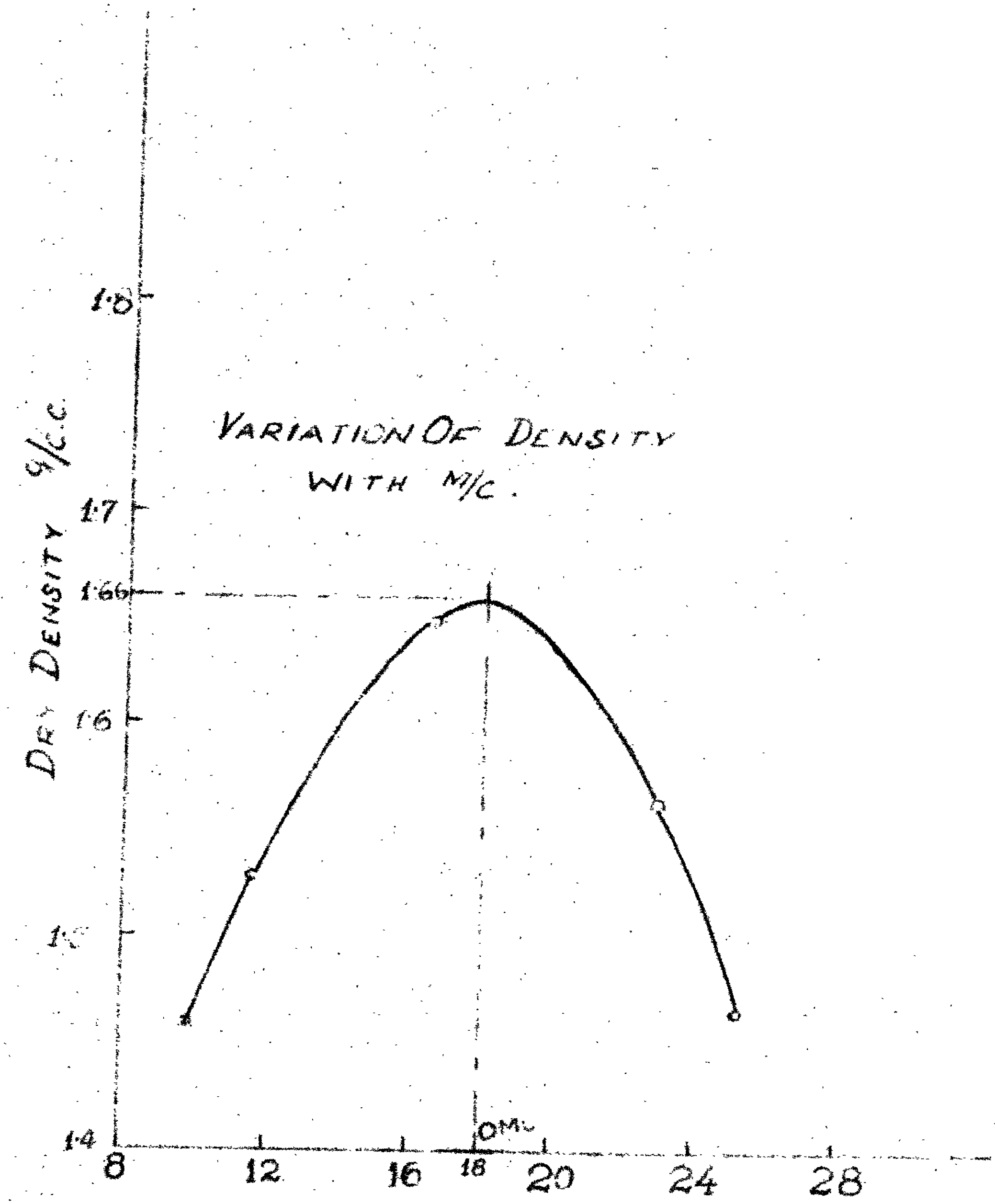
To the best of author's knowledge, no published work, except of Kondner (1962), on the line of present work, is available to data. However the importance of the present work lies in the fact, that it simulates the field loading to a better degree. More emphasis has been given in finding out the resonant frequencies of the samples and its effect on the compressive strength of specimen.

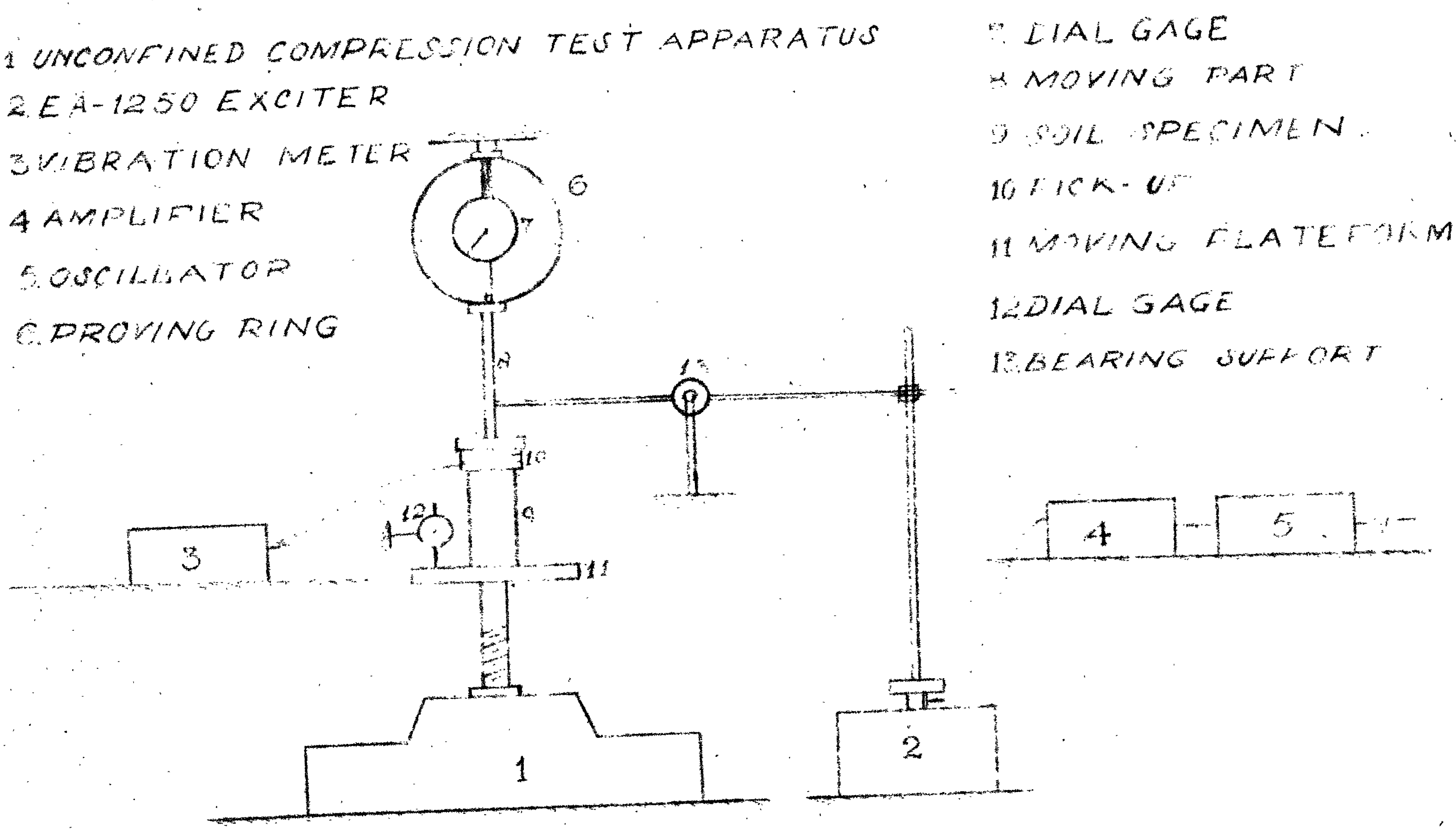
GRADATION CURVE



PARTICLE SIZE IN MM

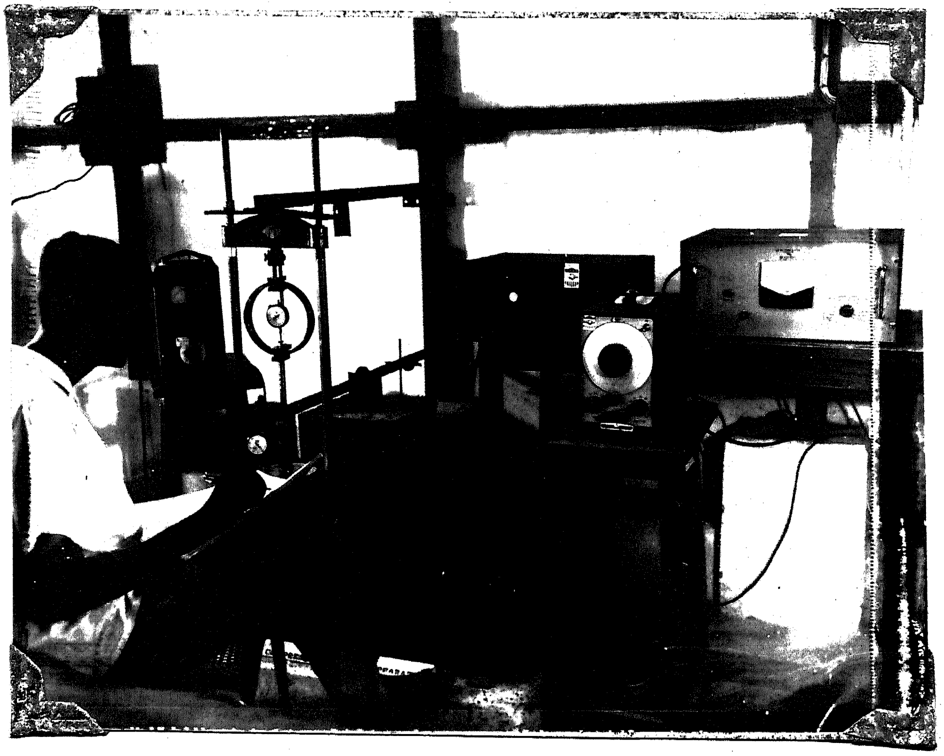
FIG 1


VARIATION OF DENSITY WITH M/C.
DRY DENSITY g/c.c
1·8
1·7
1·66
1·6
1·4
8
12
16
18
20
24
28
OMC



VIBRATORY UNCONFINED COMPRESSION APPARATUS
(NOT TO SCALE)

FIG. 3



EXPERIMENTAL SET UP FOR UNCONFINED COMPRESSION UNDER VIBRATORY LOAD

CHAPTER III

EXPERIMENTAL STUDY:-

Soil Type:- The soil selected for the observation is a silty clay, obtained in the vicinity of I.I.T.Campus. The grain size distribution curve of the soil is shown in fig.1. The soil type is CI (Silty clay) according to the Indian standard on classification of soils. The constituents are :-

Sand	5%
Silt	75%
Clay	20%

Other pertinent properties are as follows:-

Specific gravity	2.65
Optimum moisture content	18%
Maximum dry density	1.66 gms./c.c.
Uniformity Cocft.	4.0
LL	37.2%
PL	21%
SL	14%
PI	16.2

The variation of dry density with moisture content is shown in fig. 2.

INSTRUMENTATION :-

The whole setup employed in carrying out the investigation is shown in fig. 3. It consists of the following:

1. Unconfined compression test apparatus:-

All the tests were conducted with a motorised unconfined compression tester. It is a constant strain device. The rate of strain was kept at 0.03 inches per minute, throughout.

2. Exciters[1] (EA 1250):-

The basic system includes a vibramate vibration exciter Electronic Power amplifier and oscillator.

The performance of the vibramatic system is as follows:-

Frequency Range	:	5-10,000 cps.
Usable to	:	20,000 cps.
Force Vector	:	25 Lbs
Displacement Limit	:	0.5" D.A.

Model 2120 MB Amplifier:-

This amplifier is used to drive the MB model EA 1250 exciter to 25 lbs rated force.

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1 MB Electronics, A Div. of Testron Electronics, INC. 781 Whalley Ave., New Haven, Conn. 06508.

200 CD WIDE RANGE OSCILLATOR:-

This serves as the source of sinusoidal signal. The frequency range is from 5 cps to 600 K cps.

MB Vibration meter (Model M 6) :-

It provides a mean of measuring vibration in units of velocity, displacement and acceleration. These units can be read directly and rapidly over a wide range with high degree of accuracy. It is designed for use with all types of MB vibration pick ups.

TEST PROCEDURE:

The whole of the study has been done in two parts:-

In part I, frequency- displacement study has been made with the samples of 1.5 inch. Diameter and 3.375 high and 4.00 inches diameter and 4.5 inches high.

In part II, soil samples of 1.5 inches in Diameter and 3.375 inches in height have been sheared to failure, in unconfined compression test apparatus, at various frequencies ranging from 0 to 30 cps.

In the test procedure arrangement was made to introduce vertical vibrations only . However the precision of the instrument was not of the degree to produce only vertical vibrations. Some other modes of vibrations were also introduced alongwith the vertical vibrations. Also there developed a small eccentricity in placing the sample.

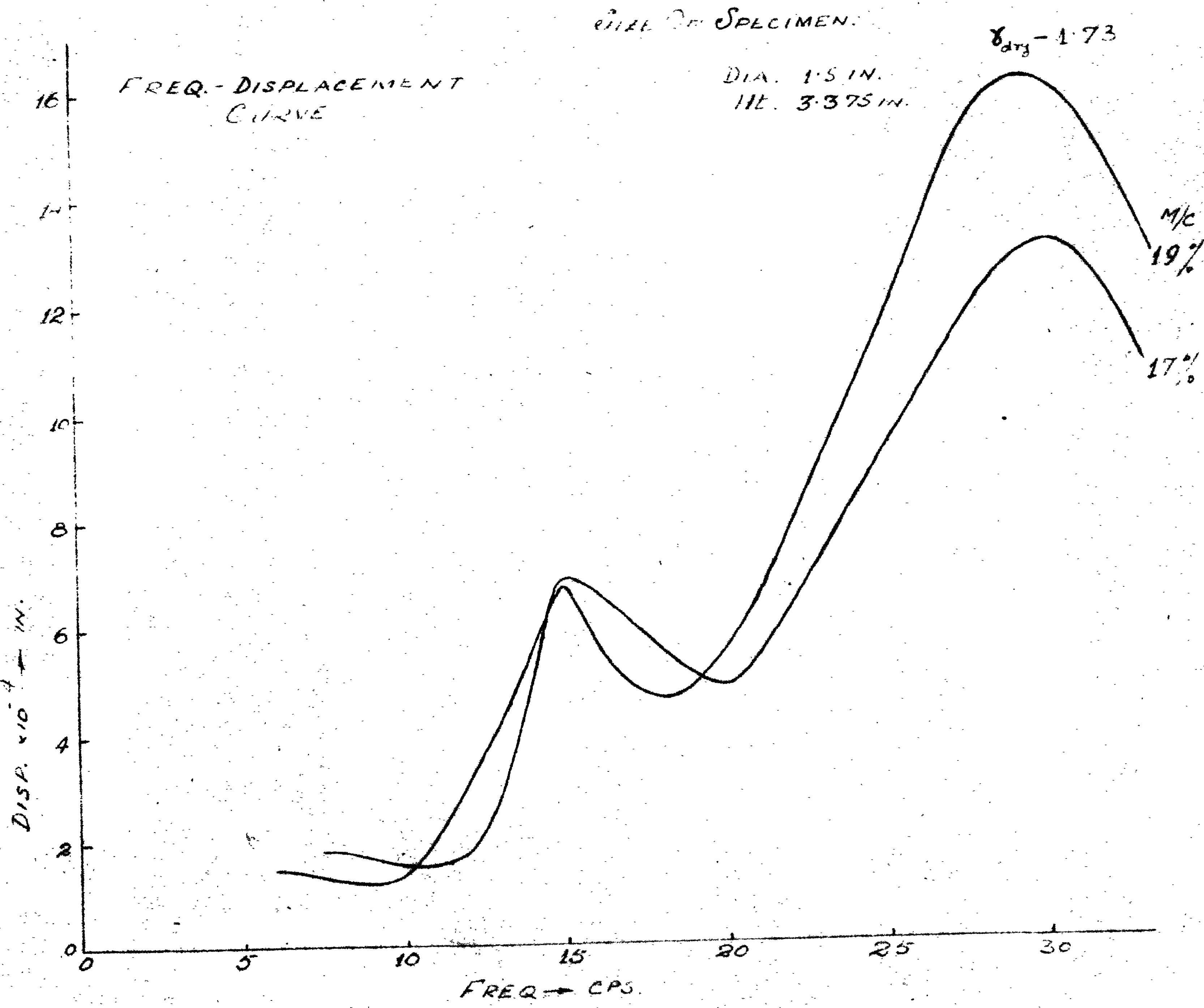

FREQ. - DISPLACEMENT CURVE
SIZE OF SPECIMEN.
DIA. 1·5 IN.
HT. 3·375 IN.
γ_{dry} - 1·73
M/C
19%
17%
DISP. $\times 10^{-4}$ → IN.
16
12
10
8
6
4
2
0
5
10
15
20
25
30
FREQ → CPS.

FIG. 4(a)

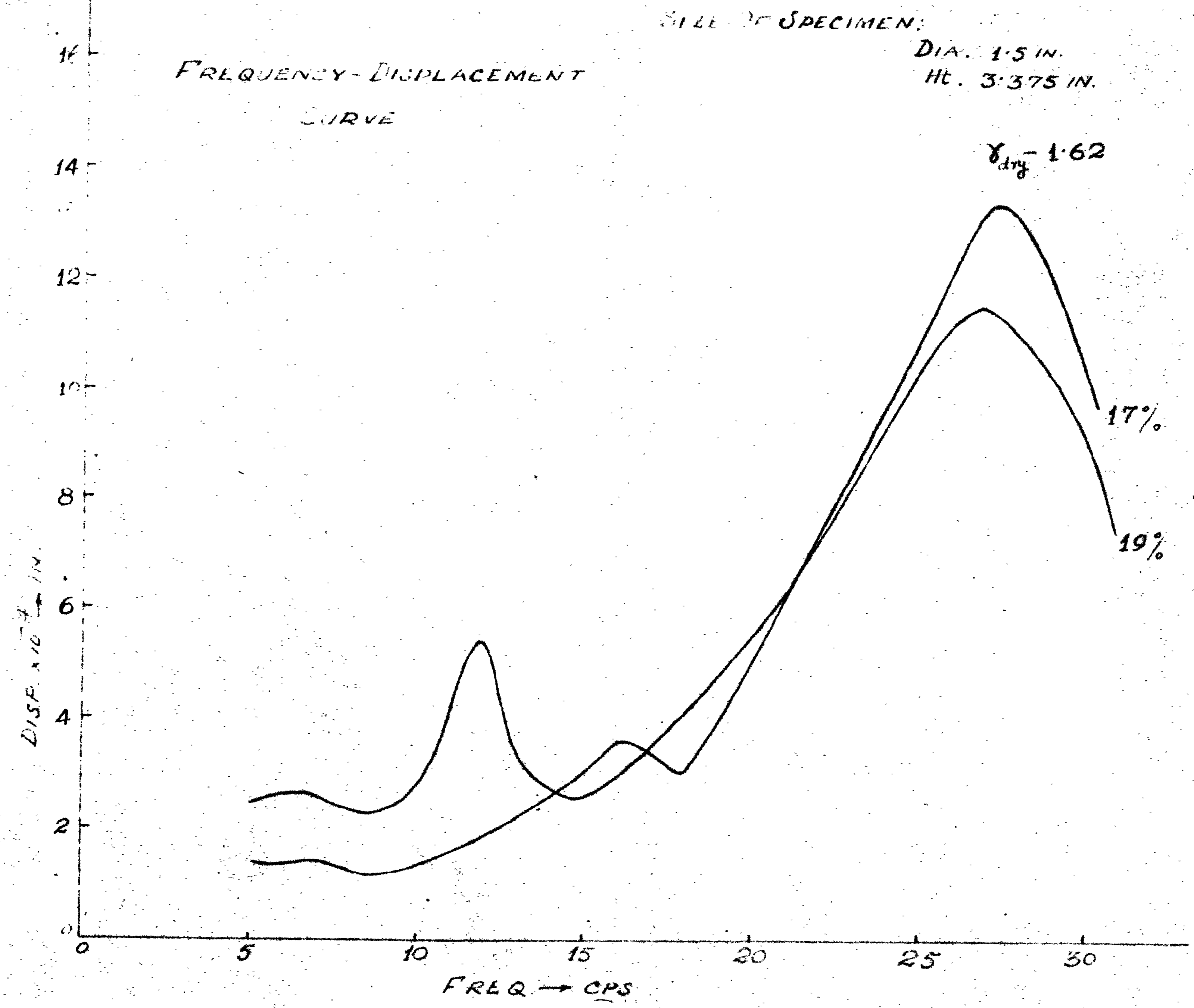

FREQUENCY - DISPLACEMENT CURVE
SIZE OF SPECIMEN:
DIA. 1·5 IN.
Ht. 3·375 IN.
γ_{dry} - 1·62
17%
19%
DISP $\times 10^{-4}$ IN
FREQ → CPS

FIG. 4(b)

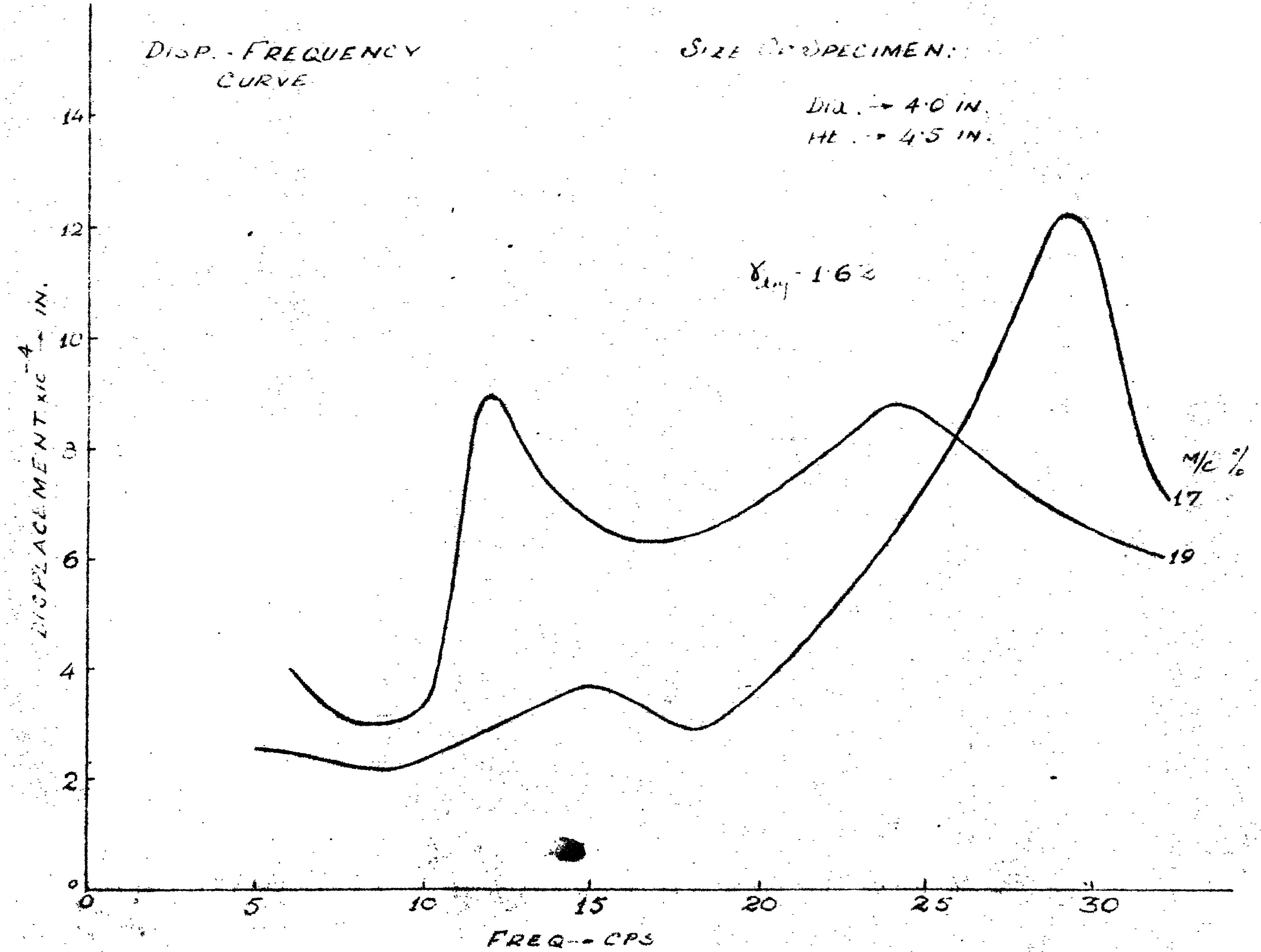
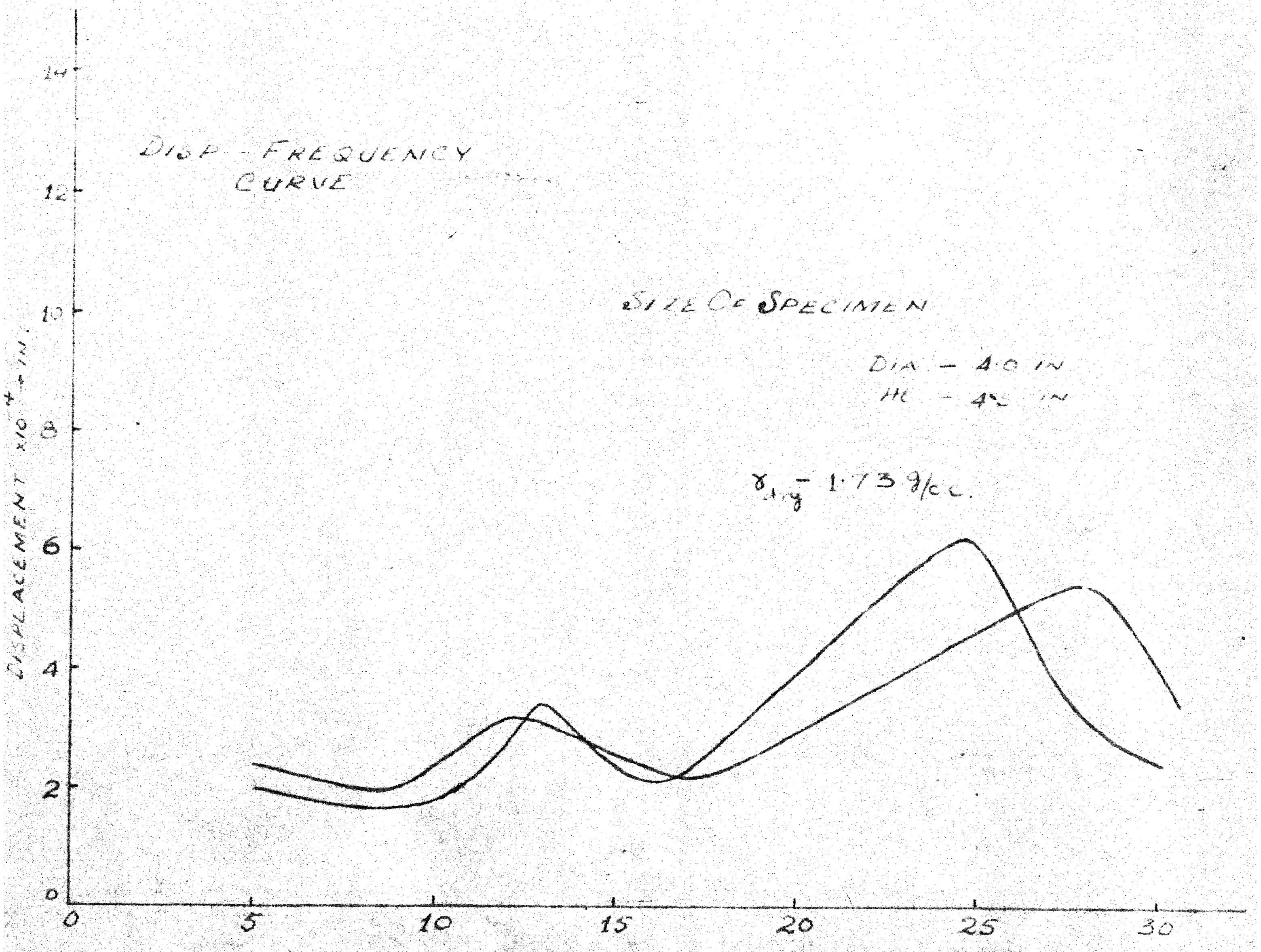

DISP. - FREQUENCY CURVE
SIZE OF SPECIMEN:
DIA. → 4.0 IN.
HT. → 4.5 IN.
γ_{dry} - 1.62
M/C %
17
19
DISPLACEMENT x10^{-4} - IN.
FREQ. - CPS
0
2
4
6
8
10
12
14
5
10
15
20
25
30

FIG. 4(c)



DISP - FREQUENCY CURVE
SIZE OF SPECIMEN
DIA - 4.0 IN
HT - 4.5 IN
γ_{dry} - 1.73 g/cc.
DISPLACEMENT x10^{-4} IN.
0
2
4
6
8
10
12
14
0
5
10
15
20
25
30
FREQ. → CPS

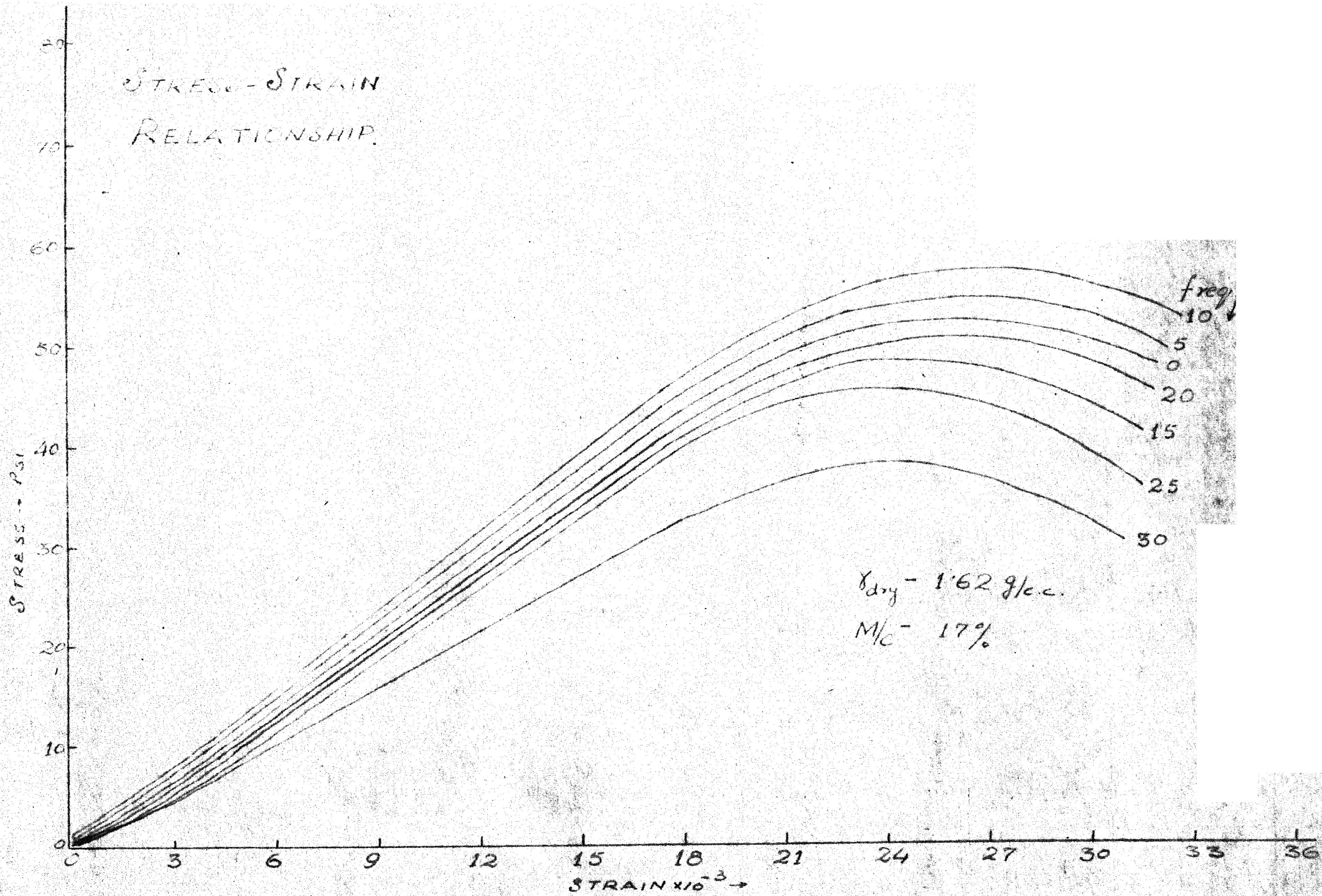
FIG. 4(d)

PART I :-

Soil samples of 1.5 inch in Diameter and 3.375 inches high were made by constant height sampling device. In this method the sampling tubes of 1.5 inch. diameter is filled with the calculated amount of soil. For a given dry density, calculated amount of dry soil is mixed with the calculated water to get required moisture content. This soil is filled in the sampling tube and compressed to a known height. This sample is extracted into a specimen mould by means of extractor and cut to the standard size.

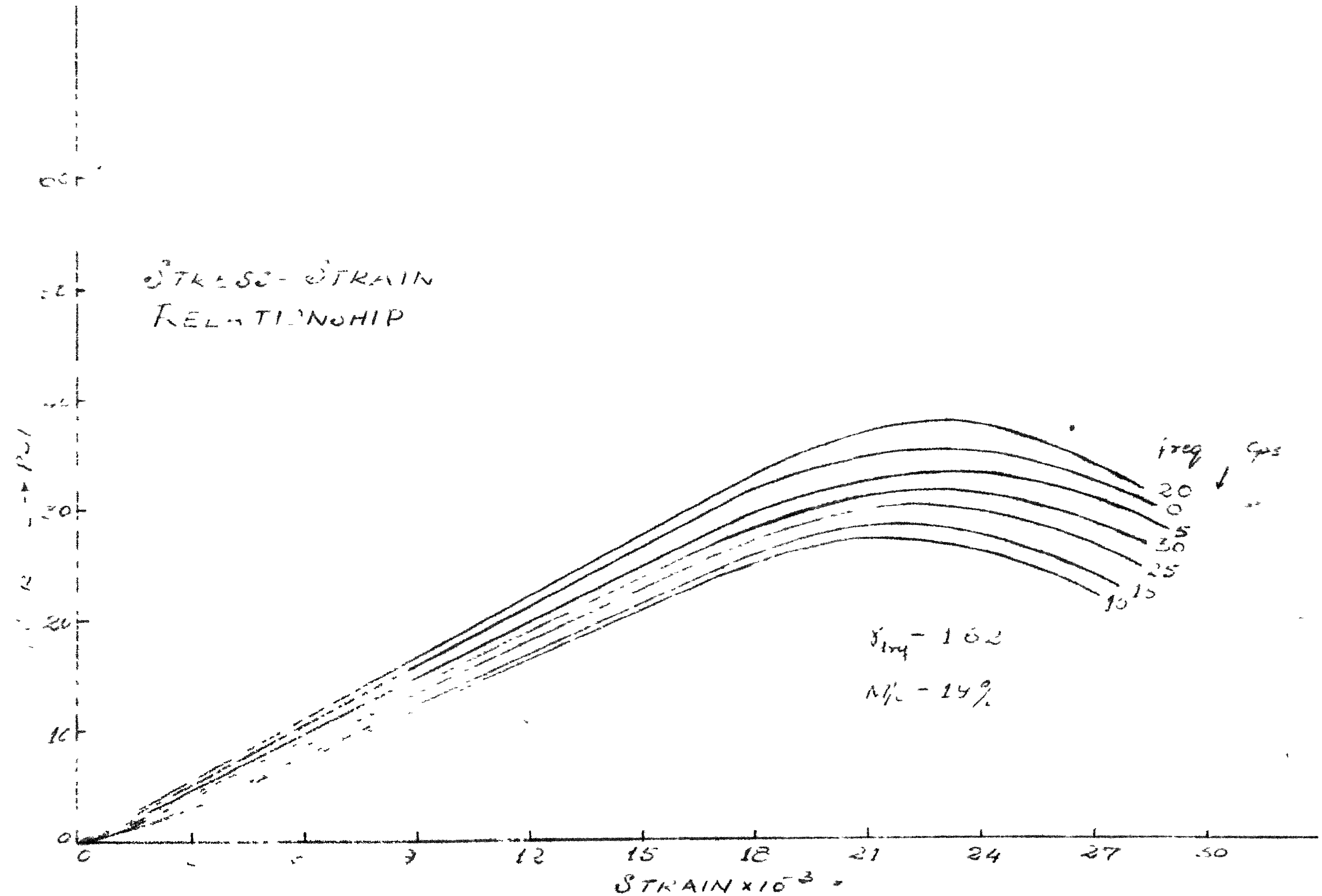
During the test the initial height of the sample was kept 4.0 inches, which was cut to the standard size of 3.375 inches in the specimen moulds. The sample of 4 inches diameter and 4.5 inch height were prepared in the proctor mould by compressing the calculated amount of dry soil, with required amount of water, by static compression. Soil samples were kept on plateform of unconfined compression test apparatus. At the upper surface of sample, pick up was placed and vibrations of constant amplitude were introduced. Keeping the amplitude of vibrations same displacements were measured by MB vibration meter, at various frequencies varying from 5 cps to 30 cps.

The results have been plotted in figure 4 (a,b,c, & d).



STRESS-STRAIN RELATIONSHIP.
freq.
10
5
0
20
15
25
30
γ_{dry} - 1.62 g/c.c.
M/C - 17%
STRESS → PSI
0
10
20
30
40
50
60
70
80
0
3
6
9
12
15
18
21
24
27
30
33
36
STRAIN $\times 10^{-3}$ →

FIG. 5(a)


STRESS - STRAIN
RELATIONSHIP
STRAIN x10^-3
freq cps
20
0
5
30
25
15
10
0
10
20
30
9
12
15
18
21
24
27
30
FIG. 5(b)

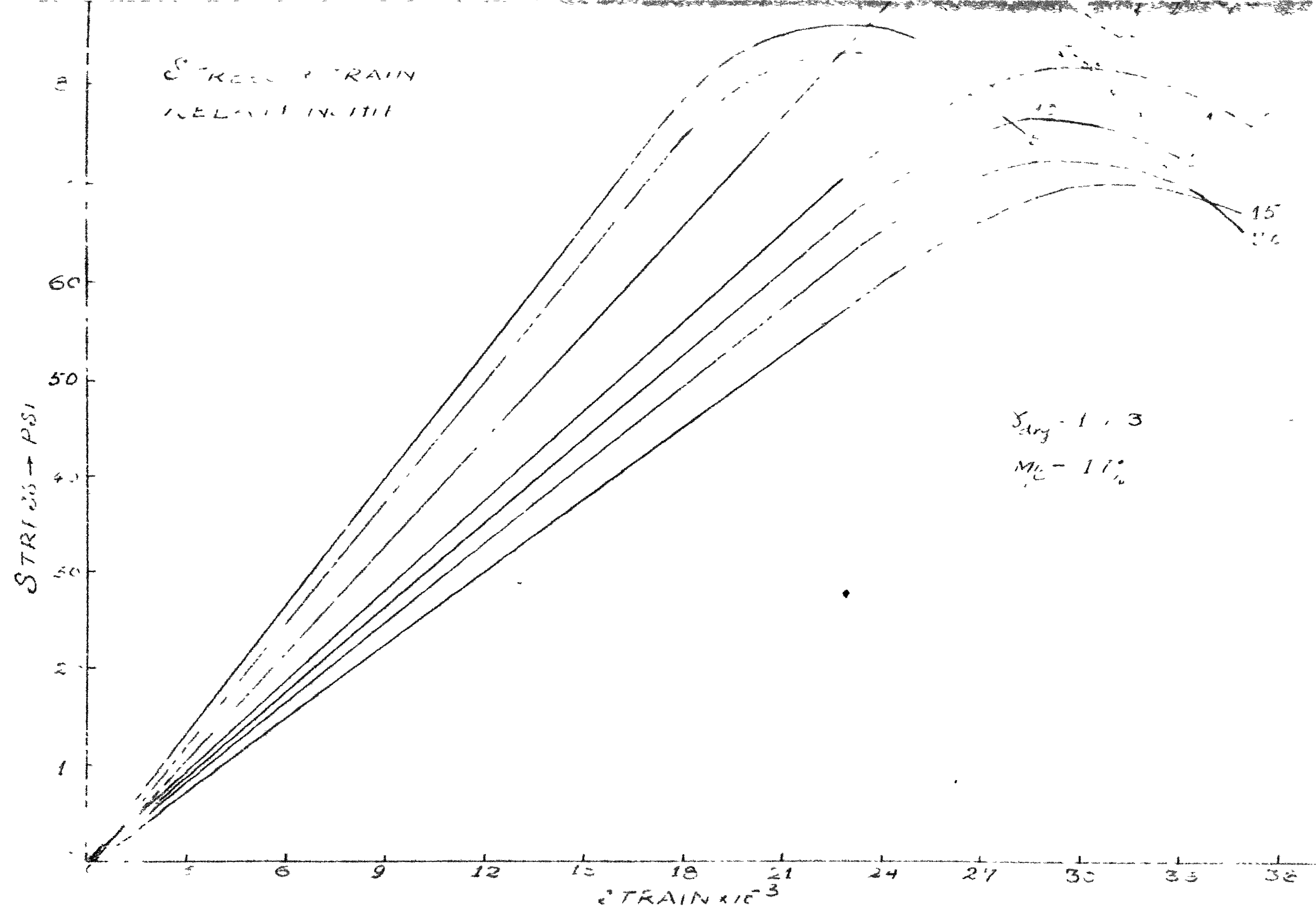


Fig 5(c)

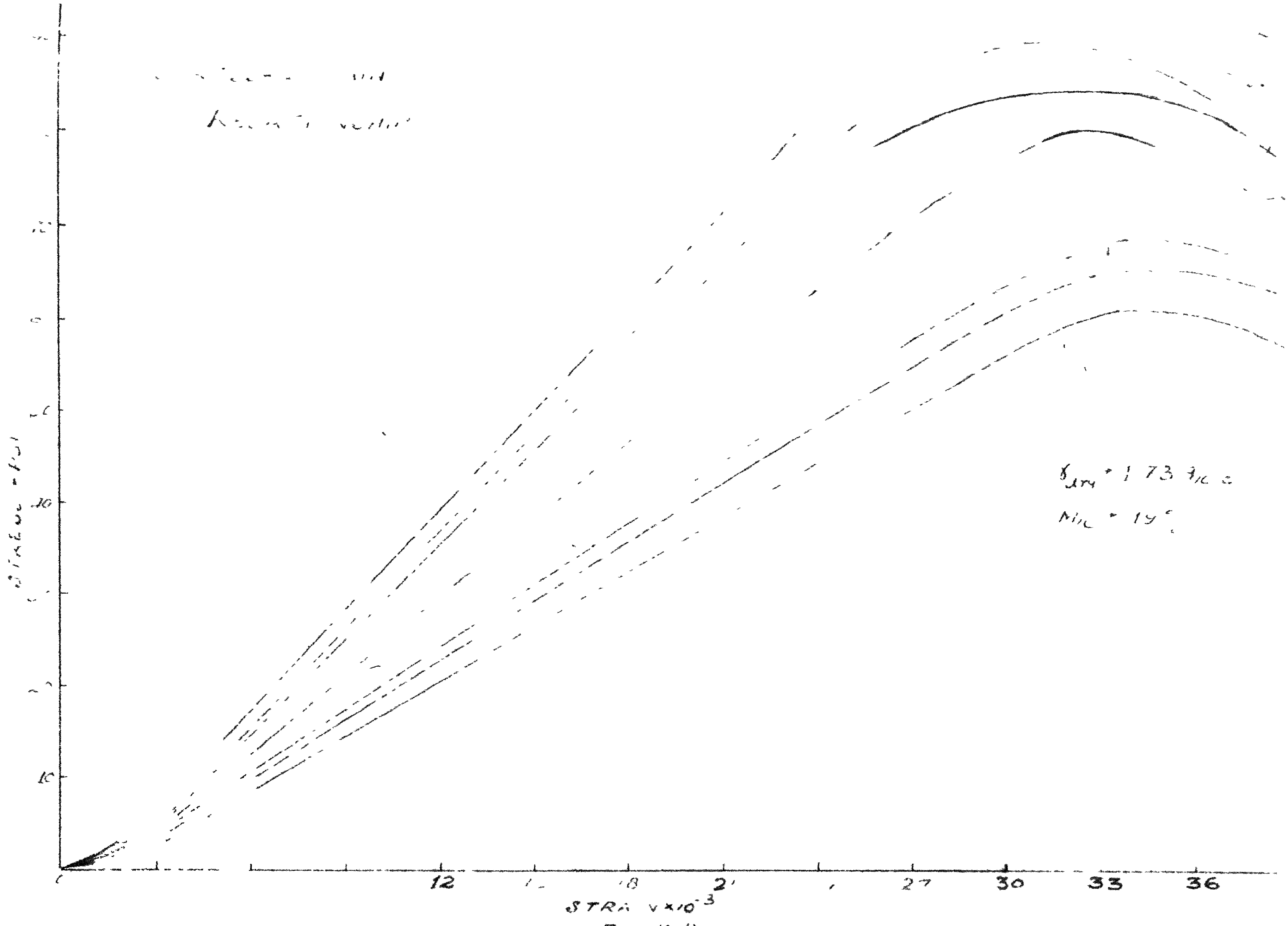


Fig 5.(d)

PART II :-

The samples of Dia. 1.5 in. and height 3.375 in. were placed on the moving plateform of unconfined compression test apparatus which was incorporated with a standard electronic vibration exciter, as shown in figure 3. The specimens were statically loaded till failure occured. The rate of deformation was maintained at 0.03 inch per minute. The loading was accompanied by small vibratory strains. The specimens were failed at various frequencies, however the amplitude of vibration of the excitor was kept constant. The study was made with the samples of dry densities 1.62 and 1.73 gms/c.c. at moisture contents of 17% and 19%.

The load-deformation relationship is shown in fig. 5 (a,b,c, & d).

VARIATION OF STRENGTH AND DISPLACEMENT
WITH FREQUENCY

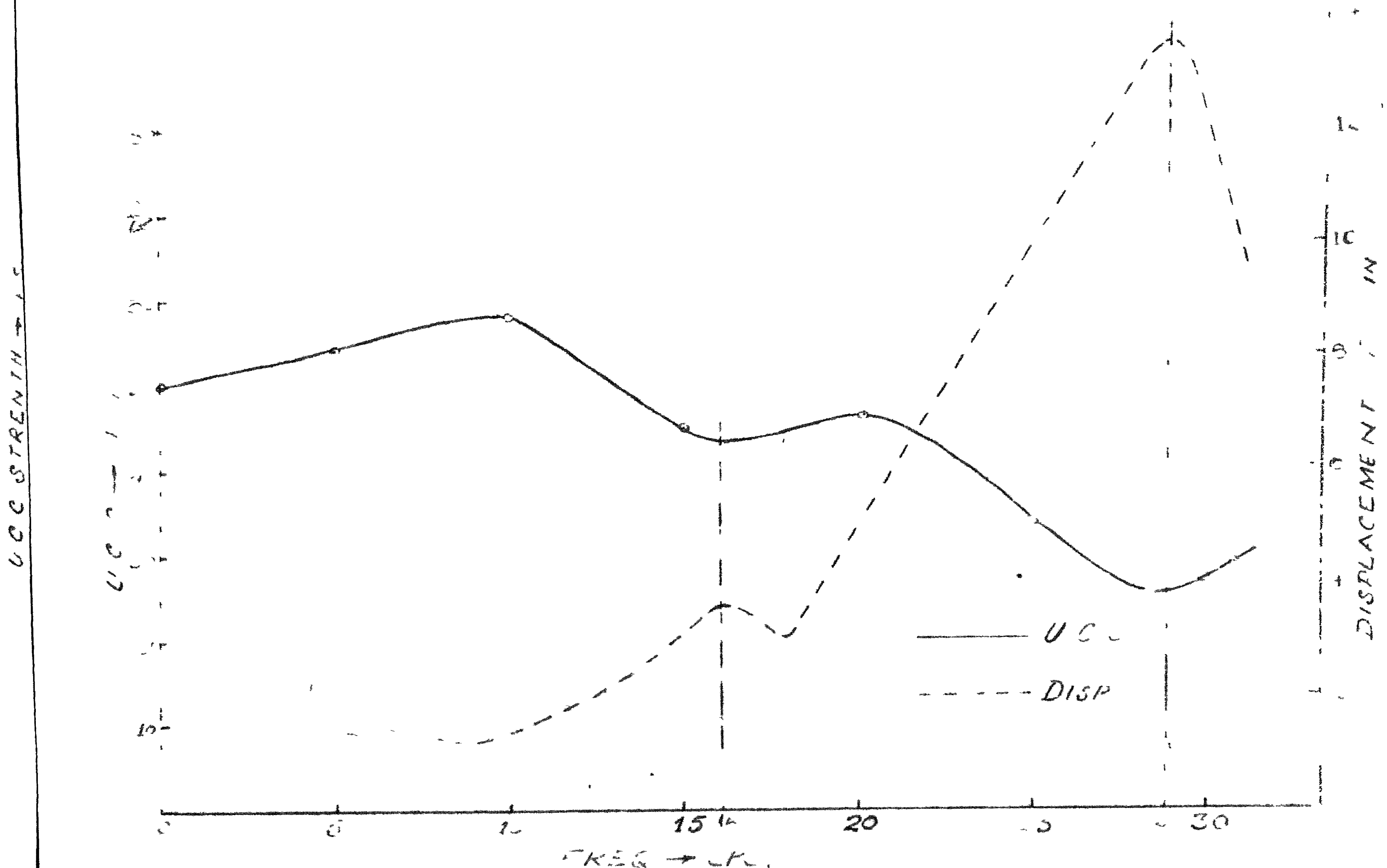
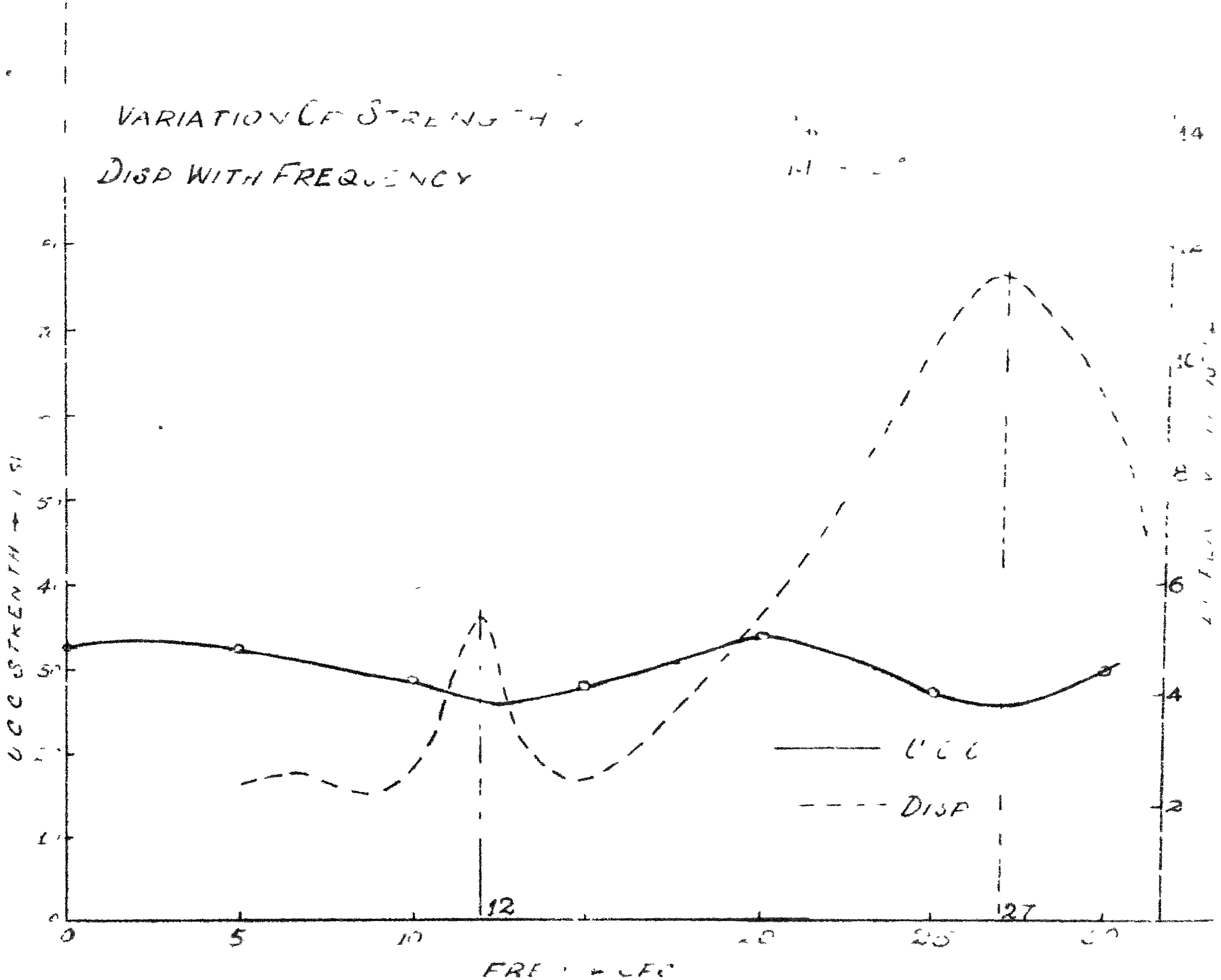
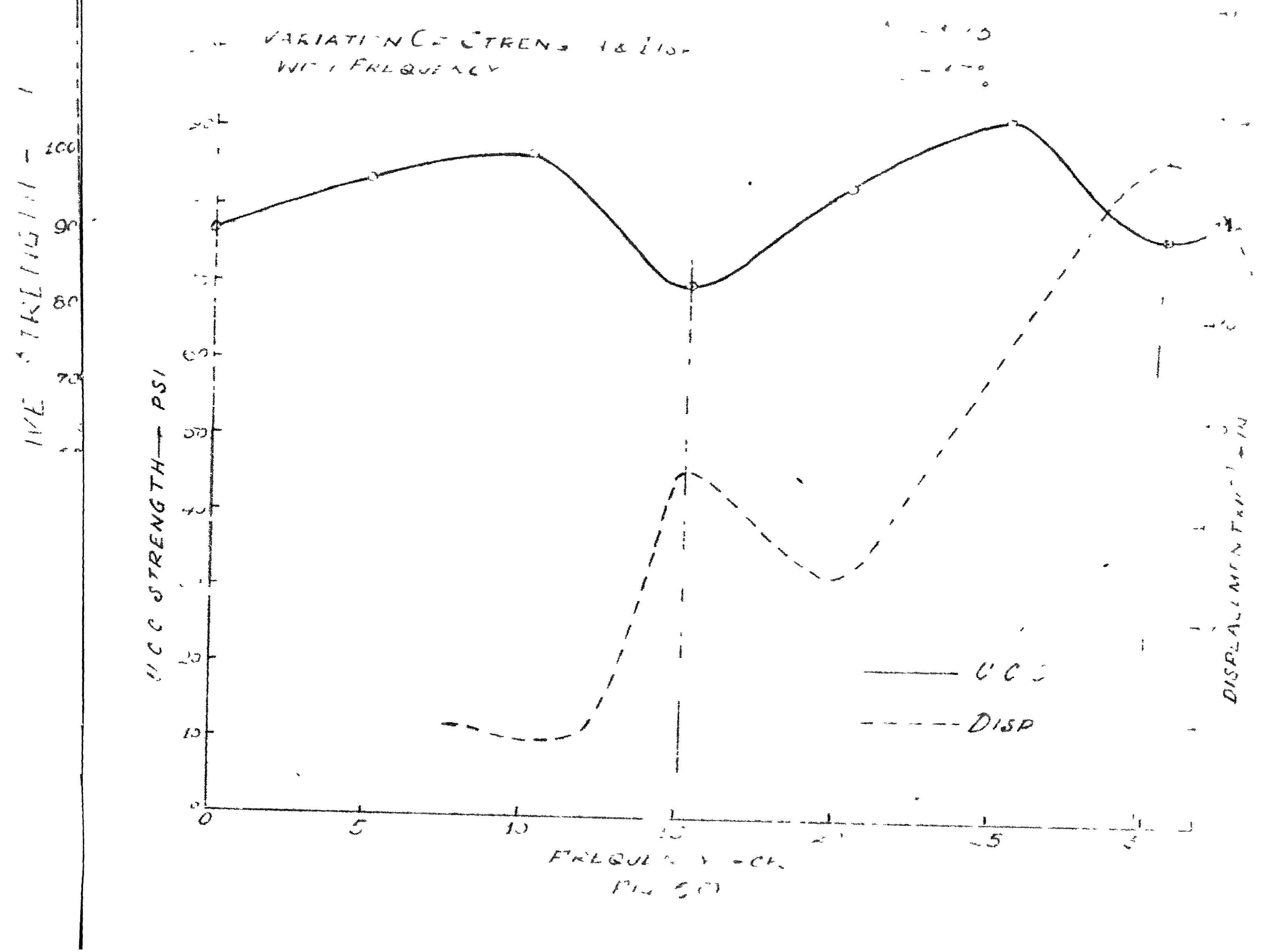


FIG. 6(a)



VARIATION OF STRENGTH
DISP WITH FREQUENCY
12
27
5
14
6
4
2
UCC
DISP

Fig 6(b)



WITH FREQUENCY
STRENGTH — PSI
DISP
0
5
60
90
80
70
100

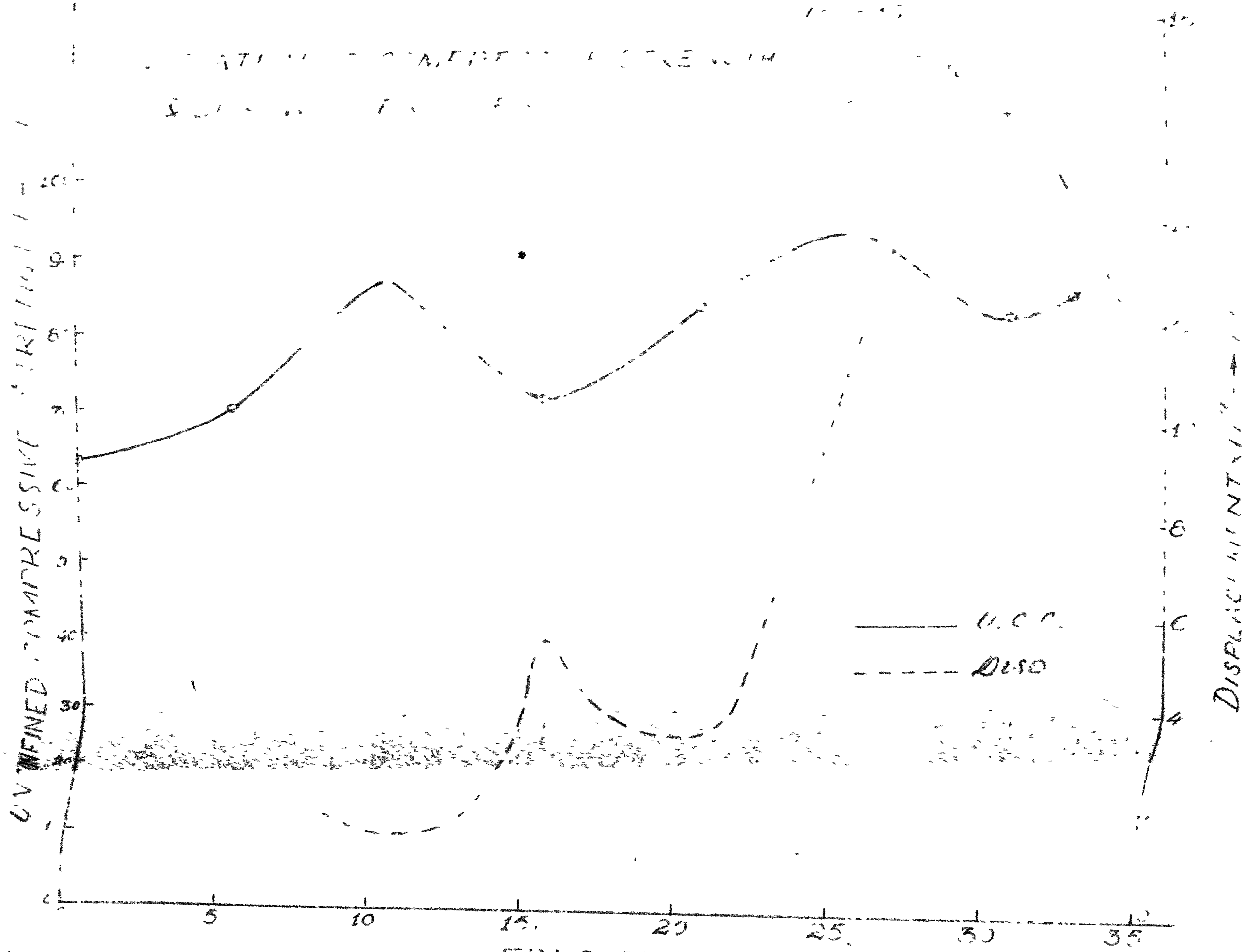

U.C.P.
DISP.
FREQUENCY → CPS
DISPLACEMENT
40
30
5
10
15
20
25
30
35
4
6
8

FIG 6(d)

CHAPTER IV

RESULTS AND CONCLUSION

RESULTS:

From the frequency-displacement curves fig. 4, it is seen that with in the range of tests, two very well defined peaks are observed. The peaks are due to two modes of vibration. The first peak occurs between 12 to 16 cps. and the second one occurs between 26 to 30 cps. This shows the occurrence of two resonant frequencies.

The displacement values, obtained with the samples of diameter 4.0 inch and height 4.5 inches, are also shown in fig. 4. And it is seen that in such cases the resonant frequencies lie in the same range as that obtained with samples of diameter 1.5 inch and height 3.375 inches.

The compressive strengths obtained from the conventional unconffined compression tests are shown in fig.6. It is seen that the unconfined compressive strength is minimum at resonant frequencies. The decrease in strength at resonant frequencies is possibly caused by the maximum amplitudes of displacement at resonant frequency. The statement is supported by the frequency displacement relation.

The effect of vibrations under frequencies not close to resonant frequency is of compactive nature because of smaller amplitudes of displacements, and , as the frequency approaches the resonant frequency, the

dilatory effects at higher amplitudes reduce the compressive strength.

Maximum stress causing failure was developed at about 3 to 4% deformation of the sample.

The modulus of deformation (slope of load-deformation curve) depends upon the maximum stress causing failure. Higher the failure stress, higher is the modulus of deformation.

From the figure 5 it is seen that unconfined compressive strength of the soil at any frequency increase with the increase in density.

The results obtained in the vibratory tests differ from the results obtained by Kondner, who finds a considerable reduction in strength of soil under vibratory loading than under static loading. Kondner's results have their own limitations, which requires modification. Kondner has not mentioned anything as to what happens to strength of soil at its resonant frequency. In the present work, the variation of compressive strength of the soil under vibratory loading has been studied with relation to its resonant ~~xx~~ frequency.

CONCLUSIONS:-

The results reported here in on the vibratory unconfined compression testing of a cohesive soil are qualitative in nature. Further investigations must be conducted before a unified test program can be evolved for formulation of quantitative conclusions inter-relating such factors as frequency of vibration, amplitude of axial strain, amplitude of axial stress, static stress level, load history, moisture content and type of soil. However the initial results indicate that such testing methods may be of considerable use in the determination of the following:

1. The degree of sensitivity of clays.

2. The resonant frequencies of the cohesive soils.

For the range of tests conducted on the clay under consideration, the following results have been conducted:

1. The strength of cohesive soil, under vibratory loading depends upon the frequency of vibration. The decrease or increase in strength depends upon the magnitude of frequency in relation to the resonant frequency. However, the results differ from that of Kondner's studies which shows considerable decrease in strength under vibratory tests as compared to conventional unconfined compression test.

2.The response of the material is frequency dependent.

3.The values resonant frequencies of the soil specimen of the sizes, 1.5 in. diameter , 3.375 in. height and 4.0 in. diameter and 4.5 in. height are of the same order .

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